

Deep soil mixing used to reduce embankment settlement

D. T. BERGADO,* T. RUENKRAIRERGS[†], Y. TAESIRI[†] and A. S. BALASUBRAMANIAM*

*Asian Institute of Technology, Bangkok, Thailand; [†]Bureau of Materials Research and Development, Department of Highways, Sri Ayuthaya Road, Bangkok, Thailand

Severe settlement and stability problems characterize the problems of the 55 km Bangna–Bangpakong Highway in Thailand. To rehabilitate this major arterial road, the deep mixing method (DMM) with soil–cement using ordinary Portland cement has been utilized for foundation improvement. In the laboratory, trial mix designs were used with a specified unconfined compressive strength of 1000 kPa. In the field, the unconfined compressive strength of the cement piles was specified at 600 kPa. Detailed soil investigations and subsequent evaluations for Section 3 from km 28 + 000 to km 34 + 500 were done. Analyses were performed regarding the bearing capacity, total settlements and their rates, and stability analyses. The predicted vertical and horizontal deformations were compared and generally agreed with the corresponding values observed in the field.

Keywords: deep soil mixing

Introduction

The Bangna–Bangpakong Highway, a 55 km long major arterial road connecting the Bangkok metropolis to the eastern seaboard of Thailand, experienced severe settlement and stability problems of the road embankment on soft Bangkok clay (Cox, 1981; Bergado *et al.*, 1990). The construction and postconstruction settlement rates in the soft areas were typically high, at 400 to 700 mm/year. The correlation between subsidence settlement and piezometric drawdown has been established by AIT (1982), which also indicated that the Bangna area is one of the worst-subsiding areas in Bangkok, with a maximum subsidence rate of 100 to 150 mm/year. In a case study involving the behaviour of Bangna–Bangpakong Highway by Bergado *et al.* (1990), the strength and compressibility parameters were confirmed by back-analyses and several methods of settlement prediction were verified. The deep mixing method (DMM) with soil–cement has been implemented for the rehabilitation of Bangna–Bangpakong Highway using ordinary Portland cement (Ruenkairergsa, 1998). Trial mix designs were used in the laboratory with a specified unconfined compressive

En Thaïlande, l'autoroute Bangna–Bangpakong, longue de 55 km, affiche de sérieux problèmes d'affaissement et de stabilité. La réhabilitation de cet axe principal a nécessité le recours à une méthode de mélange en profondeur (DMM) de sol et de ciment Portland ordinaire afin d'en améliorer les fondations. En laboratoire, des essais sur échantillons de différentes concentrations en ciment ont été réalisés à des forces de compression non confinée de 1000 kPa. Sur le terrain, les forces de compression non confinée ont été fixées à 600 kPa. Des recherches détaillées sur le sol et des évaluations des résultats sur la section 3 (entre le km 28 + 000 et le km 34 + 500) ont été réalisées. Des analyses portant sur la capacité de support, sur l'affaissement total et relatif, ainsi que sur la stabilité, ont été effectuées. La comparaison des résultats a montré que les déformations verticales et horizontales attendues concordent avec les valeurs observées sur le terrain.

strength of 1000 kPa. The details of the DMM methods are as follows: diameter $d = 60$ m, centre-to-centre spacing $S = 1.50$ m and length of cement pile = 14 m and 16 m for the km 14 + 000 to 20 + 000 and km 20 + 000 to 35 + 000 sections, respectively. The embankment height is specified at 2.5 m. The unconfined compressive strength of the cement piles in the field was specified at 600 kPa. A cement content of 150 kg/m³ with water–cement ratio of 1.5:1 was mostly used in the wet mixing method in the field. The rehabilitation scheme is shown in Fig. 1. Detailed soil investigations and subsequent evaluations for Section 3, from km 28 + 000 to km 34 + 500, were done. Analyses were made regarding the bearing capacity, total settlements and their rates, and stability analyses. Finally, the predicted vertical and horizontal deformations were compared with the corresponding values observed in the field.

Soil profile and soil properties

The borehole locations and soil profile of Section 3, from km 28 + 000 to km 34 + 500, of the Bangna–Bangpakong Highway Rehabilitation Project are given in Fig. 2. A total of 13 boreholes were made, with depths ranging from 22.95 to 35.00 m. The field test included standard penetration tests (SPTs) in the stiff clay and dense sand layers. The laboratory tests included water content, Atterberg limits, sieve analyses,

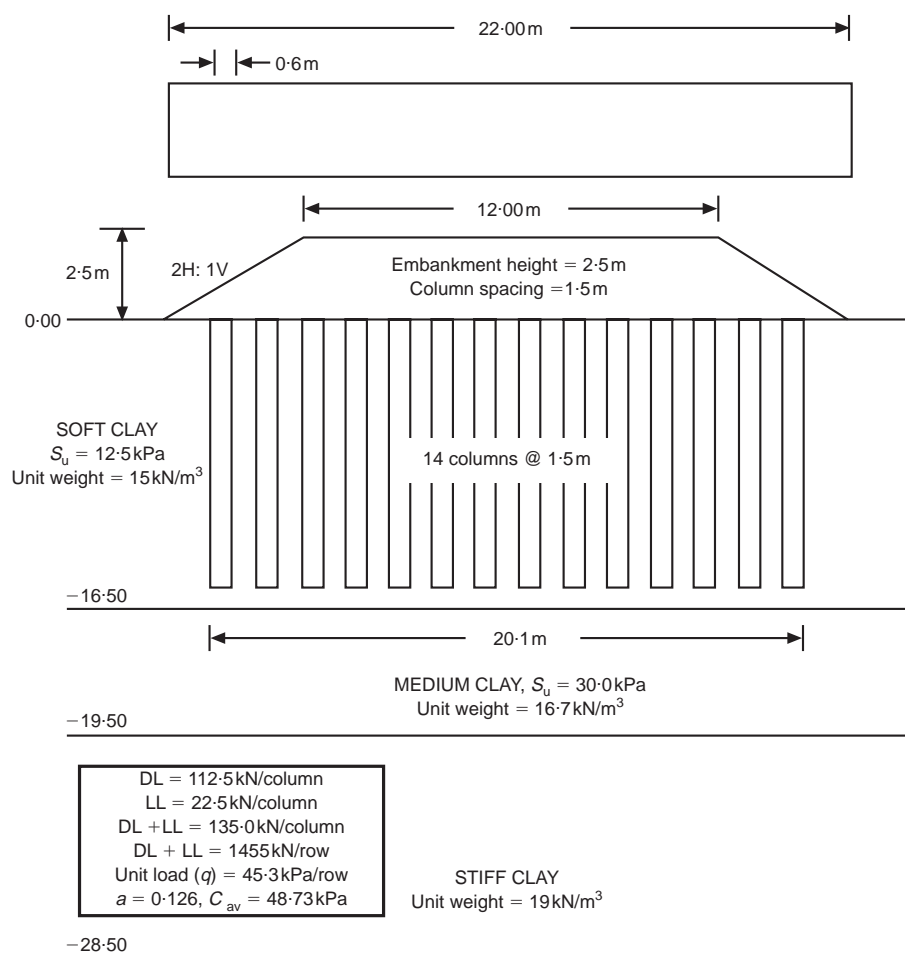


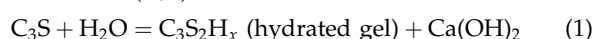
Fig. 1. Rehabilitation scheme for Bangna-Bangpakong Highway using DMM ground improvement

unconfined compression and oedometer tests. The generalized soil profile obtained from the boring logs is also shown in Fig. 2. The soil strata can be classified into several layers. The weathered crust with sand fill forms the topmost 2 to 3 m thick layer, with a brownish colour and a typical undrained strength of 30 kPa. The underlying soft clay layer extends to a depth of 16 to 18 m and is dark grey in colour. The typical natural water content ranges from 60 to 130%, the total unit weight from 14 to 17 kN/m³, the liquid limit from 70 to 120% and the plastic limit from 20 to 45%. The typical undrained shear strength varies from 10 to 25 kPa. The medium-stiff clay layer, greyish in colour with fine sand lenses, extends from about 18 to 22 m depth. The typical values of undrained shear strength vary from 25 to 50 kPa. The greyish-brown stiff clay extends from 22 to 30 m depth. The SPT *N* value ranges from 10 to 20 blows per foot. The medium dense to dense, brownish, silty sand extends to the end of the borings. The SPT *N* values range from 20 to 60 blows per foot.

In the soft clay layer along the Bangna-Bangpakong Highway, Balasubramaniam and Bergado (1984) reported organic contents ranging from 2 to 5% with an occasional maximum value of 9%. Uddin (1995) reported organic contents of 5.6%. Balasubramaniam *et al.* (1985) reported salt contents of 0.50 to 2% (5 to 20 g/l). The groundwater level was located at depths ranging from 0.70 to 1.90 m. The undrained shear strengths from unconfined compression tests are plotted against depth in Fig. 3. A summary of soil parameters for the settlement analyses is given in Table 1.

Fundamental concept of clay-cement stabilization

Type I Portland cement is widely used in soil stabilization. According to Lea (1956), the four major strength-producing compounds of Type I Portland cement are tricalcium silicate (C₃S), dicalcium silicate (C₂S), tricalcium aluminate (C₃A) and tetracalcium aluminoferrite (C₄AF). When the pore water in the soil encounters the cement, hydration of the latter occurs rapidly. The major hydration (primary cementitious) products are hydrated calcium silicates (C₂SH_x, C₃S₂H_x), hydrated calcium aluminates (C₃AH_x, C₄AH_x) and hydrated lime Ca(OH)₂. The first two of the hydration products are the main cementitious products. In addition, the hydration of cement increases the pH because of the dissociation of hydrated lime. Consequently, the strong base dissolves the silica and alumina (which are inherently acidic) from both the clay minerals and the amorphous materials of the clay particle surfaces, in a manner similar to the reaction between a weak acid and a strong base. The hydrous silica and alumina will then gradually react with calcium ions, liberated from the hydrolysis of cement, to form insoluble compounds (secondary cementitious products) which harden when cured, to stabilize the soil. This secondary reaction is known as the pozzolanic reaction. For instance, the reactions involving tricalcium silicate (C₃S) can be as follows:



(primary cementitious product);

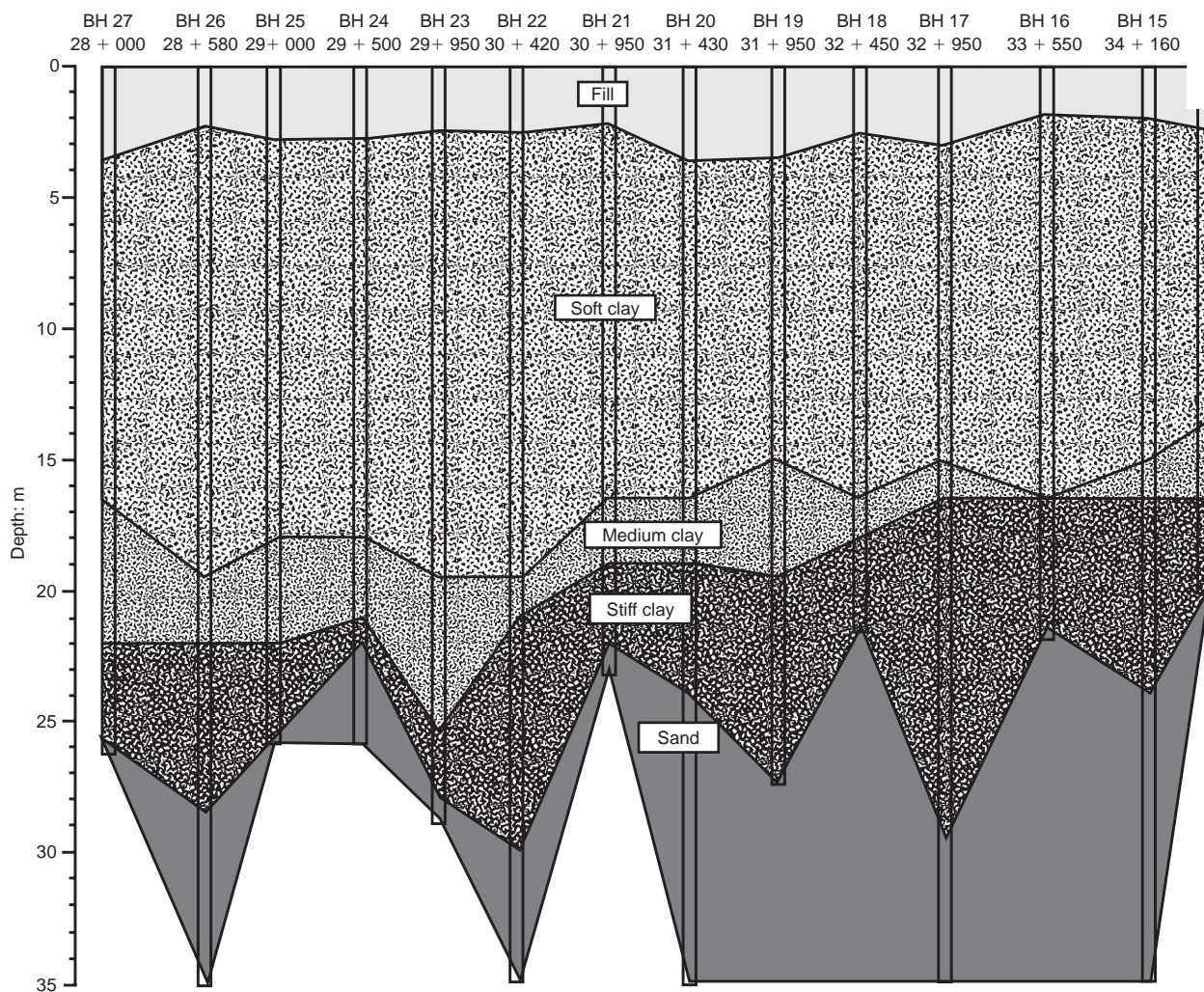
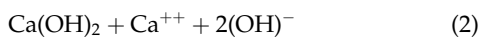
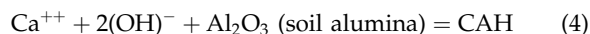


Fig. 2. Soil profile along station STA-28 + 000 to 34 + 500, Bangna-Bangpakong



(secondary cementitious product);



(secondary cementitious product).

The cement hydration and pozzolanic reaction can last for months, or even years, after mixing. Thus, the strength of cement-treated clay tends to increase with time.

Lime/cement piles

Lime/cement piles were developed in Sweden in the 1970s. Deep mixing methods were also developed in Japan in the 1970s. Both lime and cement additives were frequently used, depending on their cost and availability. The most common applications of lime/cement piles are for improving the stability and reducing the settlement of highway embankments, small, flexible buildings, tanks and other lightly loaded, small structures. For deposits of clays of 15 to 20 m thickness, the entire depth can be stabilized. Detailed information on lime/cement piles can be found in Holtz (1989), Van Impe (1989), Broms (1984, 1993), Bergado *et al.* (1996), Schaefer (1997) and Munfakh (1997).

Trial mix design

One representative borehole from Section 3 of the Bangna-Bangpakong Highway was selected for the trial mix tests of the cement pile. Soft clay samples at depths of 3, 6, 12 and 15 m were taken and the undrained shear strength was determined by the unconfined compression test. Soft clay samples from each depth were intimately mixed with cement at a content of 125 to 250 kg/m³ of wet soil. After mixing and moulding, the stabilized soil was cured for 3, 7, 14 and 28 days. At least three specimens were moulded for each curing period. The moulding of the specimen was done in such a way that a constant unit weight for each cement content was obtained. The average undrained shear strength of the specimens for each curing period were determined by unconfined compression tests. An undrained shear strength of 500 kPa at 28 days was used to determine the amount of cement for use in cement pile installation. However, in any case the amount of cement used in the field should not be less than 150 kg/m³ of wet soil. The results of the trial mix design for Section 3 of the Bangna-Bangpakong Highway are shown in Fig. 4.

Cement pile installation

Cement piles were made by the *in situ* mixing of the foundation clay and cement to form cement piles having a

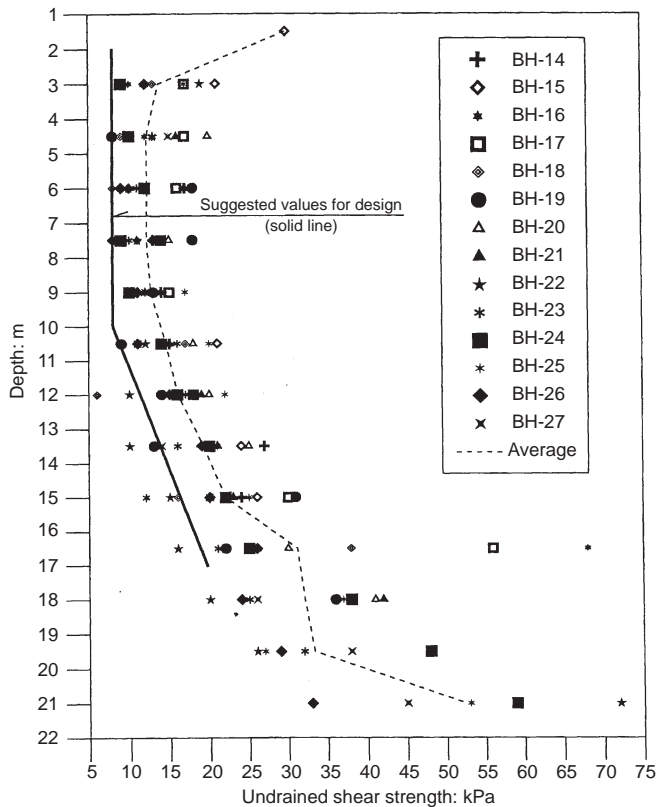


Fig. 3. Undrained shear strength from unconfined compression test in Section 3

diameter and depth as specified. *In situ* wet mixing was adopted. For the wet mixing process the equipment included a cement powder storage silo, a water supply tank, a water pressure pump, a slurry mixing chamber with a metering system, a cement slurry storage tank and a cement slurry pump injector for mixing the subsoil with cement at depth. The measuring devices required for monitoring and control were those for slurry mix content, slurry grouting pressure and flow rate, rotation speed and withdrawal rate. In mixing the subsoil with cement slurry, the injection system and leading rod must have the capability to induce a consistent and homogeneous soil–cement mix *in situ*. The length of the leading rod and injection nozzle assembly should be enough to produce the specified length of cement pile. The slurry mix had a water/cement ratio of 1.5:1. The system for cement pile installation is shown in Fig. 5. The DMM machine and cement pile installation are shown in Fig. 6.

Quality control of cement piles

Quality control of the cement piles during construction was performed for every 3000 piles installed as follows: (a) a test to determine the undrained shear strength from an

unconfined compression test, and (b) a test to determine the dimensions of the pile. If the test pile failed to conform to the specified value, a pile load test would be conducted on a nearby pile. If the load-carrying capacity of the test pile was not less than 200 kN, then all the cement piles installed could be accepted. In addition, the water–cement ratio of the cement slurry would be checked from time to time to be sure that the amount of cement in the slurry mix was adequate. The variation with depth of the undrained shear strength at 28 days of the cement piles for all four sections of the Bangna–Bangpakong Highway that employed the wet mixing process and a cement content of 150 kg/m³ are plotted in Fig. 7. The undrained shear strength of the cement piles tends to vary from 300 to 600 kPa, with most values falling between 300 and 400 kPa. The minimum specified value of 300 kPa was satisfied.

Ultimate bearing capacity of single cement piles

The bearing capacity of a single cement pile is governed either by the shear strength of the surrounding soft clay (soil failure) or by the shear strength of the cement pile (pile failure). The short-term ultimate bearing capacity of a single pile in soft clay, assuming soil failure, can be calculated from the following equation by Broms (1984, 1993):

$$Q_{ult,soil} = (\pi d H_{col} + 2.25\pi d^2) C_u \quad (5)$$

where d is the diameter of the pile, H_{col} is the pile length and C_u is the average undrained shear strength of the surrounding soft clay. For C_u values less than 30 kPa, the skin friction has been found to be equal to the undrained shear strength. The point resistance is assumed to be $9C_u$.

For the case where the bearing capacity is governed by cement pile failure, the behaviour of the cement pile is similar to that of stiff fissured clay. The failure occurs along the fissures of the cement pile. The short-term ultimate bearing capacity in the cement pile, assuming pile failure at depth z , can be estimated as follows (Broms, 1984, 1993):

$$Q_{ult,col} = A_{col}(3.5C_{col} + 3\delta_h) \quad (6)$$

where C_{col} is the cohesion of the cement pile material and δ_h is the total lateral pressure acting on the pile at the critical section at depth z . The angle of internal friction of the cement pile was assumed to be 30° (Uddin, 1995). The factor 3 corresponds to the value of the coefficient of passive pressure, K_p , at $\phi_{col} = 30^\circ$. The value of σ_h was assumed to be equal to $\sigma_v + 5C_u$, where σ_v is the total overburden pressure. The long-term ultimate strength may be lower than the short-term strength owing to creep. The creep strength Q_{creep} of the cement piles was assumed to be 80% of $Q_{ult,col}$.

The layout of the cement piles, at a spacing of 1.5 m for an embankment height of 2.5 m, is given in Fig. 1. The ultimate and allowable bearing capacities are given in Table 2. The short-term ultimate bearing capacities of a single

Table 1. Parameters used for settlement analyses

Depth: m	γ : kN/m	σ_{vo} : kPa	σ_p : kPa	RR	CR	E_u : kPa	C_v : m ² /yr	C_h : m ² /yr
0–3	17.5	26–25	50	0.030	0.30	2600	2.5	5.0
3–9	14	49.5	50	0.045	0.45	1560	2.0	4.0
9–14	14.5	72–75	77	0.040	0.398	1820	2.0	4.0
14–16	14.5	88.5	95	0.035	0.35	2340	2.0	4.0
16–18	15.5	98.5	113	0.030	0.30	3250	2.5	5.0
18–19.5	16.5	108–88	125	0.025	0.25	4550	2.5	5.0

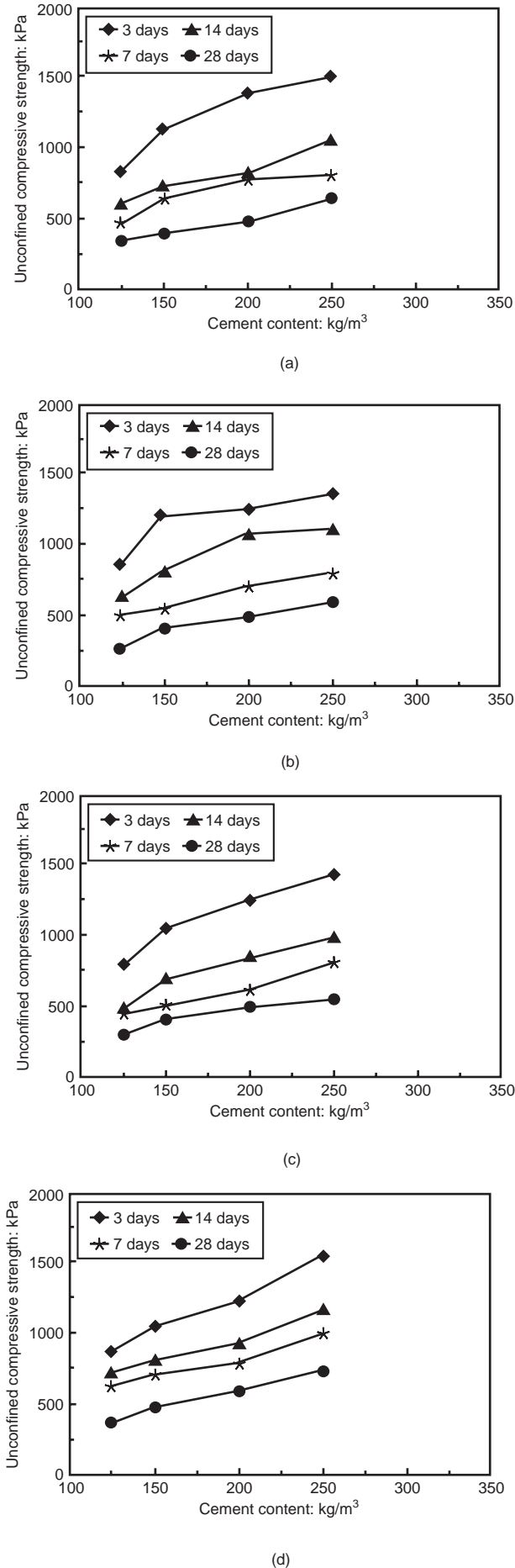


Fig. 4. Variation in strength of cement-treated clay with cement content, Bangna–Bangpakong Highway, km 29 + 500, at depths of (a) 3 m, (b) 6 m, (c) 12 m, (d) 15 m

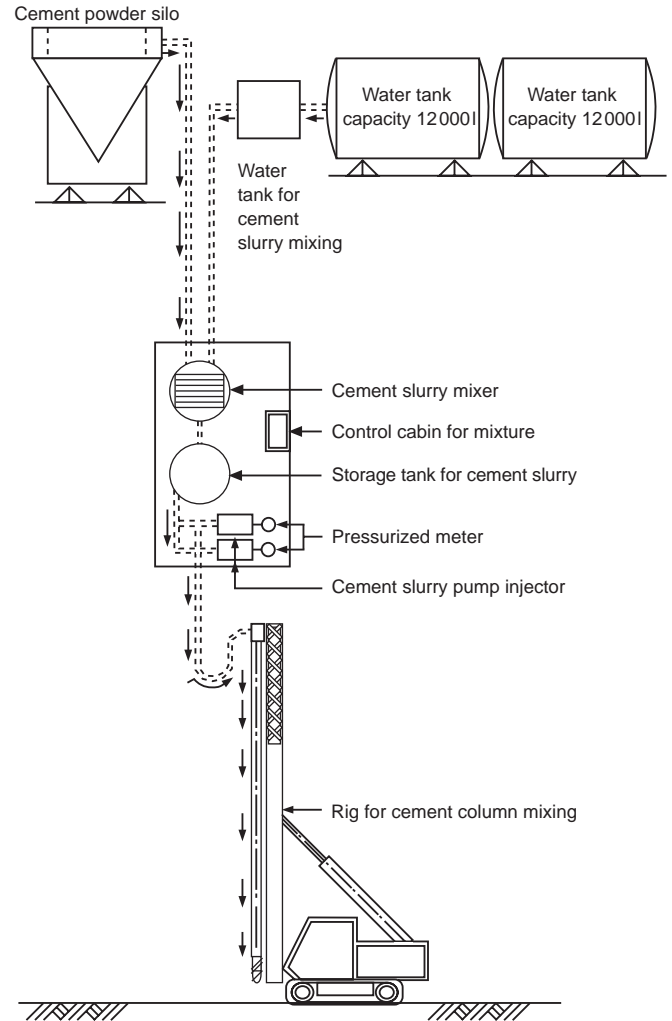


Fig. 5. System for manufacturing cement columns

cement pile assuming soil failure, $Q_{ult,soil}$, at pile tip depths of 14, 16 and 18 m were 347.2, 392.4 and 526.3 kN, respectively. The ultimate bearing capacity assuming pile failure, $Q_{ult,col}$, at the corresponding depths was 475.0 kN. The load levels are tabulated in Table 3 and the creep stresses are given in Table 4. Comparing Tables 3 and 4, the creep limit has not been exceeded. The load levels in Table 3 at a pile spacing S of 1.50 m are well below the allowable bearing capacity in Table 2 and also lower than the specified value of 200 kN from the plate bearing test in the field.

Settlement analyses

The total settlement of the untreated ground, S_t , is calculated by the conventional method as follows:

$$S_t = \sum h_i [RR_i \log(\sigma_p / \sigma_{vo}) + CR_i \log(\sigma_{vf} / \sigma_p)] \quad (7)$$

where h_i is the thickness of sublayer i ; RR_i and CR_i are the recompression and compression ratios, respectively, of layer i ; σ_{vo} is the effective overburden pressure; σ_p is the maximum past pressure; and σ_{vf} is the final effective vertical stress.

The settlement of the cement-pile-treated ground is calculated assuming equal strain between the cement piles and the surrounding ground. The increase of the vertical stress distribution in the treated ground is assumed to be uniform and is equal to the surcharge at the top of the



(a)



(b)

Fig. 6. (a) Mixing blades of DMM installation machine; (b) installation of cement piles by DMM machine

cement piles. The vertical stress increases in the untreated zone below the bottom of the cement piles are assumed to be followed by a 2 to 1 slope method. The settlement reduction ratio μ_c , which is the ratio of the total settlements down to the bottom of the treated zone with and without cement piles, can be estimated from the relationship

$$\mu_c = \frac{1}{a_s(E_{col}/E_{soil}) + (1 - a_s)} \quad (8)$$

where E_{col} and E_{soil} are the Young's moduli of the cement pile and soil material, respectively; a_s is the area replacement ratio ($a_s = (d/D_e)^2$); d is the cement pile diameter; and D_e is the equivalent diameter of the unit cell of the treated zone.

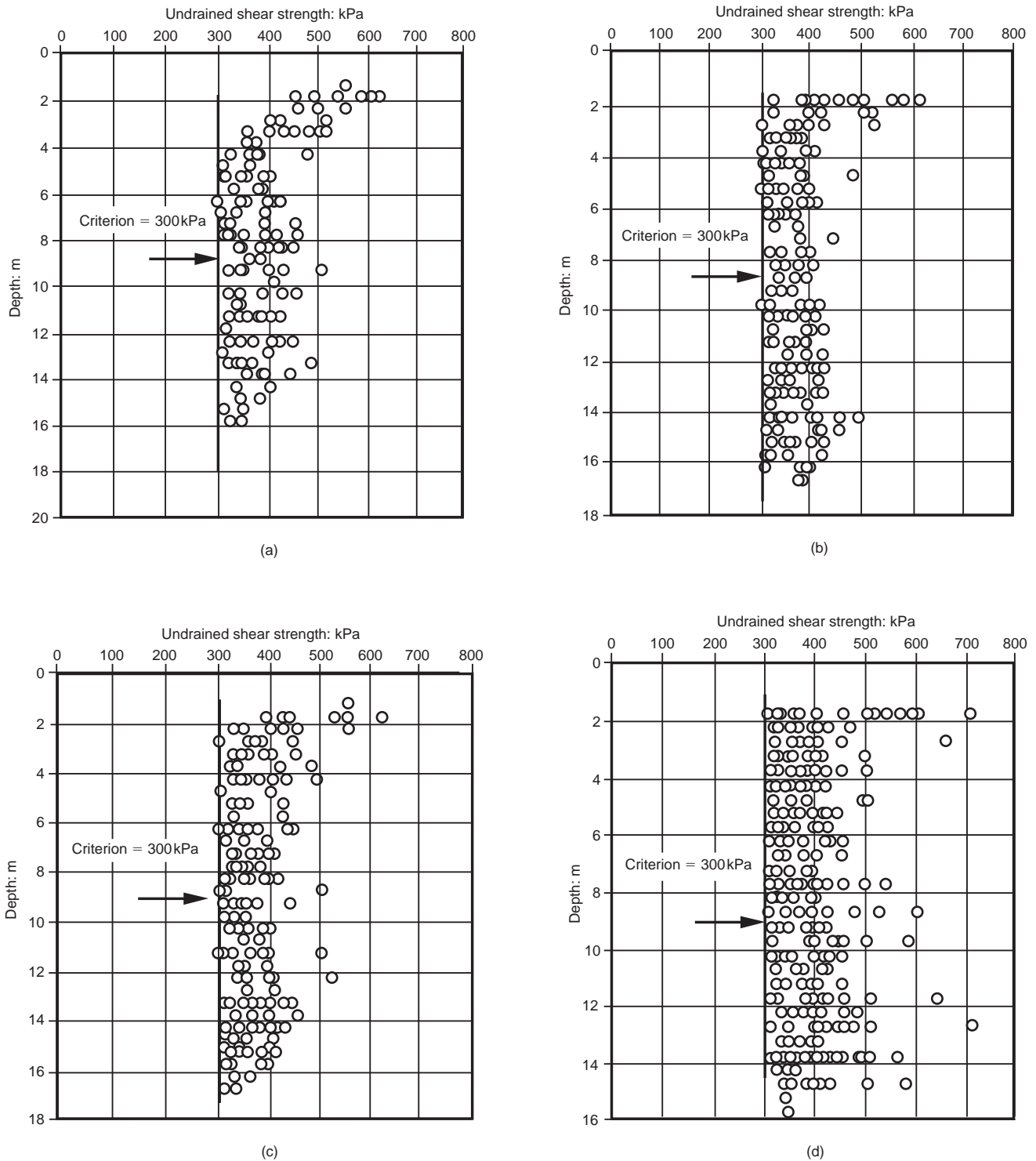


Fig. 7. Undrained shear strength of cement columns constructed on Bangna–Bangpakong Highway: (a) Section 1, (b) Section 2, (c) Section 3, (d) Section 4

Table 2. Ultimate and allowable bearing capacities

Pile tip: m	$Q_{ult, soil}$: kN	$Q_{ult, col}$: kN	Q_{all} (FS = 1.5): kN
14.00	347.2	475.0	23.15
16.00	392.4	475.0	26.16
18.00	526.3	475.0	31.7

Table 3. Load levels at embankment heights of 2.5 m and 3.0 m: kN

Spacing	Height = 2.5 m	Height = 3.0 m
1.50	135.0	157.5
1.70	173.4	202.3
1.90	216.6	252.7
2.00	240.0	280.0

The degree of consolidation U of the sublayer within the treated zone is calculated by the following equation:

$$U = 1 - (1 - U_h)(1 - U_v) \quad (9)$$

where U_v is the degree of consolidation in vertical flow only

and U_h is the degree of consolidation in radial flow only. U_h is estimated by Hansbo (1979) as follows:

$$U_h = 1 - \exp\left(\frac{-8T_h}{F}\right) \quad (10)$$

Table 4. Creep loads and creep stresses

Tip: m	$Q_{\text{creep-col}} (80\% Q_{\text{ult,col}})$	Stress _{creep-col} : kPa
14-00	38-00	1344
16-00	38-00	1344
18-00	38-00	1344

and

$$F = F_n + F_s + F_r \quad (11)$$

$$F_n = \log_e(D_e/d) - 0.75 \quad (12)$$

$$F_s = 0 \quad (13)$$

$$F_r = \pi z(2L - z)(K_h/q_w) \quad (14)$$

where z is the distance from the point considered to the drainage boundary; L is the pile length; K_h is the radial permeability of the surrounding soil; and q_w is the discharge capacity of the pile, calculated as follows:

$$q_w = K_{\text{col}} \left(\frac{\pi d^2}{4} \right) \quad (15)$$

where K_{col} is the permeability of the cement pile material. Equations (10) and (11) lead to the following expression:

$$F_r = \frac{4z(2L - z)}{d^2} \left(\frac{K_h}{K_{\text{col}}} \right) \quad (16)$$

The settlement of the treated zone S_t at time t is estimated from the corresponding value for untreated ground, S_{0t} , through the settlement reduction ratio μ_c as follows:

$$S_t = \mu_c S_{0t} \quad (17)$$

and

$$S_{0t} = U_t S_t \quad (18)$$

The settlement analyses were carried out for different values of cement pile length and spacing. The soil parameters used for the settlement analyses are given in Table 1. The Young's modulus of cement piles was assumed to be 100 times the undrained shear strength ($E_{\text{col}} = 100C_{\text{col}}$) as obtained from Uddin (1995). The value of C_{col} was 300 kPa, according to the design specifications. The permeability ratio of the surrounding soil and cement piles ($K_{\text{soil}}/K_{\text{col}}$) was taken as 20 (Uddin et al., 1997). The settlement reductions μ_c are tabulated in Table 5. The settlements of untreated ground for embankment heights H of 2.5 and 3.0 m are shown in Table 6. For the portion from km 28+000 to km 30+950 at an embankment height of 2.5 m and a cement pile length of 16.0 m, the calculated settlements for different pile spacings are plotted in Fig. 8. From Fig. 8, the settlement reduction ratio was estimated as 0.46.

Table 5. Settlement reduction ratio μ_c caused by cement pile treatment

Depth: m	μ_c^*		
	$S = 1.5 \text{ m}$	$S = 1.7 \text{ m}$	$S = 2.0 \text{ m}$
0-3	0.430	0.492	0.573
3-9	0.303	0.359	0.437
9-14	0.339	0.398	0.477
14-16	0.402	0.464	0.545
16-18	0.491	0.554	0.632
18-19.5	0.587	0.646	0.717

* Calculated with $E_{\text{col}} = 100C_{\text{col}} = 30\,000 \text{ kPa}$.

Table 6. Results of primary settlement calculation for embankment of untreated ground

Depth: m	Settlement: m	
	$H = 2.5 \text{ m}$	$H = 3.0 \text{ m}$
0-3	0.190	0.238
3-9	0.748	0.861
9-14	0.295	0.363
14-16	0.072	0.090
16-18	0.031	0.044
18-19.5	0.014	0.021
Total	1.35	1.62

Stability analyses

The undrained shear strengths used in the stability analyses are plotted against depth in Fig. 3. The stability analyses of untreated and treated ground were conducted for the most critical failure plane, assuming undrained loading at the end of construction. The stability analyses were performed using the simplified Bishop method of slices. A live load of 10 kPa was applied in the calculations. The equivalent shear strength in the treated or improved zone was estimated using the average strength method. The factors of safety are summarized and plotted in Fig. 9 for improved ground at embankment heights of 2.5 and 3.0 m. For a 2.5 m high embankment, the factor of safety for the untreated case is 0.73, while for the improved case at a cement pile spacing of 1.5 m, the corresponding factor of safety is 2.42. The values of the factor of safety are not affected by the pile lengths for pile lengths longer than 12 m.

Finite-element analysis

A 2.5 m high embankment on soft ground improved by 16 m long cement piles at 1.5 m spacing (square pattern) was modelled by the finite-element method (FEM) in the plane strain case. The cement piles were converted into a continuous wall having the same area replacement ratio. For cement piles of 0.6 m diameter at a spacing of 1.5 m, the continuous-wall thickness in the plane strain case was 0.19 m. The soft-soil model of Vermeer and Brinkgreve (1995), which resembles the modified cam-clay model, was used for the foundation soils. The soft-soil model required six parameters: the friction angle ϕ' , apparent cohesion c' , modified compression ratio λ^* , modified swelling ratio κ^* , shear modulus G and Poisson's ratio ν' . For the embankment fill, the elastic-perfectly plastic Mohr-Coulomb theory was used to simulate hard soils such as compacted soils and overconsolidated soils. The elastic-perfectly plastic Mohr-Coulomb theory required five parameters: the friction angle ϕ' , cohesion c' , dilatancy angle ψ , shear modulus G and Poisson's ratio ν' . However, the dilatancy can be assumed equal to zero for this soil. For the cement piles, an elastic model was used. The elastic model for cement piles required two parameters: the shear modulus G and Poisson's ratio ν' . The model parameters corresponding to the portion from km 28+000 to km 30+950 are presented in Table 7. The deformed FEM mesh at the end of construction is shown in Fig. 10, indicating an undrained compression in the treated case of 0.20 m. The maximum lateral displacement was obtained as 0.073 m as shown in Fig. 11. The displacement fields are plotted in Fig. 12, where rotational failure can be expected to be the most critical failure mode. The results from the fully drained case indicated a final settlement of

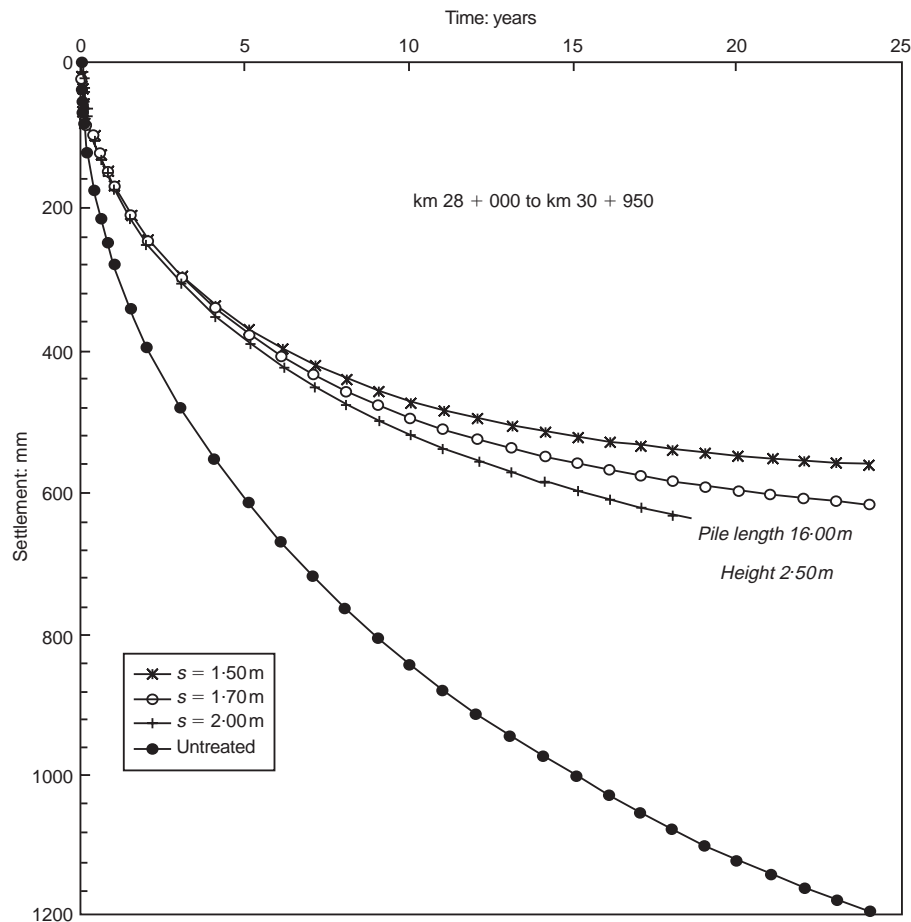


Fig. 8. Settlement-time relationship for pile tip at 16.00 m and embankment height 2.5 m with various column spacings

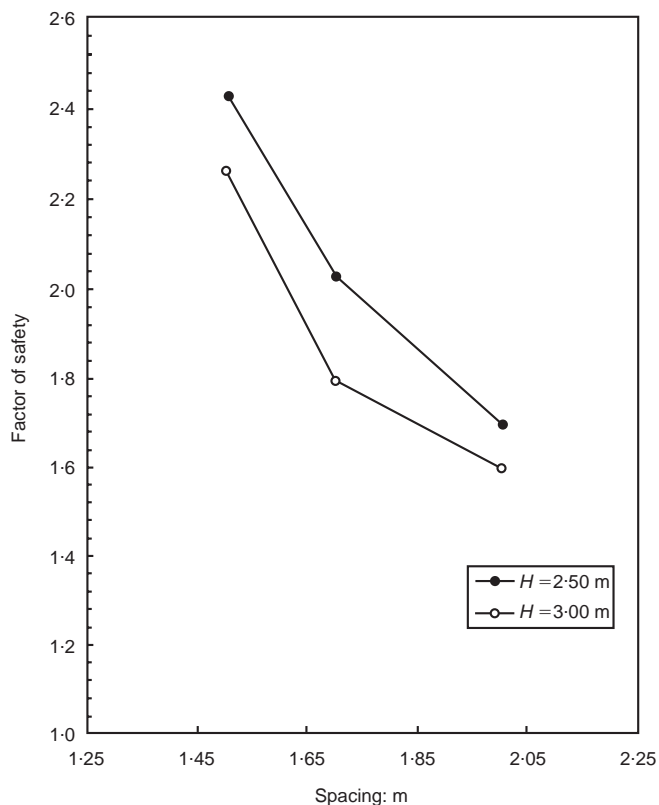


Fig. 9. Factors of safety for embankment heights of 2.5 and 3 m on cement-column-treated ground

0.38 m and 1.26 m for the treated and untreated ground conditions, respectively, as presented in Figs 13 and 14, respectively. By using cement piles of 0.6 m diameter at 1.5 m spacing, the average value of the settlement reduction ratio was obtained as 0.30.

Comparison with observed data at km 29 + 992

Field monitoring instruments were installed in both the main road (MR) and frontage road (FR) embankments. Plan and section views at km 29 + 992 are shown in Fig. 15. The instrumentation includes surface and subsurface settlement plates, piezometers, earth pressure cells and an inclinometer. An observation well was used to monitor the groundwater level. The surface settlements and earth pressure cells were placed on cement piles and between piles. The detailed locations of the surface settlement plates and earth pressure cells in the MR and FR are shown in Figs 16 and 17, respectively. The surface settlement records of the MR and FR are plotted in Figs 18 and 19, respectively. Two large settlement readings in Fig. 19 are considered outliers due to disturbance of the instruments. In general, the settlement magnitudes between and on the cement piles are similar, indicating equal strain according to the assumption of Broms (1984, 1993).

The observed surface settlements in the treated or improved zone varied from 0.15 to 0.70 m over a period of

Table 7. Soil parameters used in FEM analyses (km 28 + 000 to 30 + 950)

Material	Model	Model parameters					
		ϕ' : degrees	c' : kPa	$\lambda^{*\dagger}$	$\kappa^{*\ddagger}$	G : kPa	ν'
Subsoil	Soft-soil model						
0–3 m		20	8	0.130	0.026	—	0.33
3–9 m		22	1	0.196	0.039	—	0.33
9–14 m		22	1	0.173	0.035	—	0.30
14–18 m		22	1	0.152	0.030	—	0.25
18–22 m		23	1	0.150	0.030	—	0.25
Embankment	Elastic–perfectly plastic (Mohr–Coulomb model)	30	1	—	—	2 800	0.33
Cement pile	Elastic model	—	—	—	—	11 280	0.33

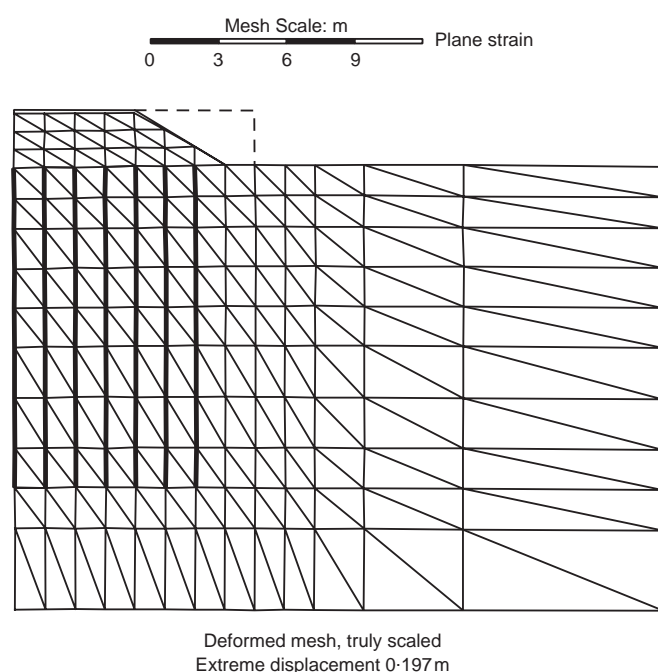
 $\dagger \lambda^* = CR/2.303.$
 $\ddagger \kappa^* = RR/2.303.$


Fig. 10. Deformed mesh for 2.5 m high embankment on treated ground—undrained analysis

about one year. Most of the observed settlements are clustered around 0.15 to 0.35 m. The predicted value from the conventional method, as shown in Fig. 8, amounted to 0.20 m at one year, which is within the range of the observed values. The predicted long-term settlement for 1.5 m spacing, from Fig. 8, is 0.56. The corresponding values from the FEM were obtained as 0.20 m and 0.38 m for the undrained and drained analyses, respectively, with a total vertical deformation of 0.58 m, which agreed with the conventional prediction. The settlement reduction ratio of 0.30 predicted from the FEM results is the lower bound of the calculated values tabulated in Table 5. The corresponding settlement reduction (Fig. 8) was calculated as 0.46, which agrees with the upper-bound values in Table 5. The observed lateral movements from inclinometer readings are shown in Fig. 20. A maximum lateral movement of 0.070 m was observed after one year. The long-term prediction from the FEM is 0.073 m, as demonstrated in Fig. 11.

Conclusions

From the results of laboratory tests and subsequent analyses, the following conclusions can be drawn.

- The bearing capacity of a single cement pile is governed either by the shear strength of the surrounding soft clay (soil failure) or by the shear strength of the cement pile (pile failure). The short-term ultimate bearing capacities, assuming soil fatigue of a single cement pile in soft clay, $Q_{ult,soil}$, at depths of 14, 16 and 18 m were 347.2, 392.4 and 526.3 kN, respectively, while the ultimate bearing capacity, assuming pile failure in the cement pile, $Q_{ult,col}$, at the same depths was 475.0 kN. These values are much above the calculated load levels as well as the specifications from the plate bearing tests.
- Stability analyses were performed using the simplified Bishop method of slices. The factors of safety for the improved ground at embankment heights of 2.5 and 3.0 m for 1.5 m pile spacing are 2.42 and 2.26, respectively. Minimum factors of safety of 1.70 and 1.60 were obtained for a pile spacing of 2.0 m at embankment heights of 2.5 and 3.0 m, respectively.
- With a 2.5 m high embankment improved with 16 m long cement piles at 1.5 m spacing in a square pattern, the observed surface settlements after a one-year period in the treated or improved zone varied mostly from 0.15 to 0.35 m, which agreed well with the value of 0.20 m predicted using the conventional method.
- Settlement analyses of treated ground were carried out for cement pile lengths of 14, 16 and 18 m at spacings of 1.5, 1.7 and 2.0 m. The 1.5 m spacing yields the smallest settlement and settlement reductions. For the portion from km 28 + 000 to km 30 + 950 at an embankment height of 2.5 m and a cement pile length of 16 m, the expected long-term settlement is 0.56 m for a spacing of 1.5 m.
- A 2.5 m high embankment on soft ground improved by 16 m long cement piles at 1.5 m spacing (square pattern) was modelled by the finite-element method in the plane strain case. The settlement values obtained from the FEM were 0.20 m and 0.38 m for the undrained and drained analyses, respectively, with a total vertical deformation of 0.58 m. This predicted value agrees with the calculated long-term settlement of 0.56 m.

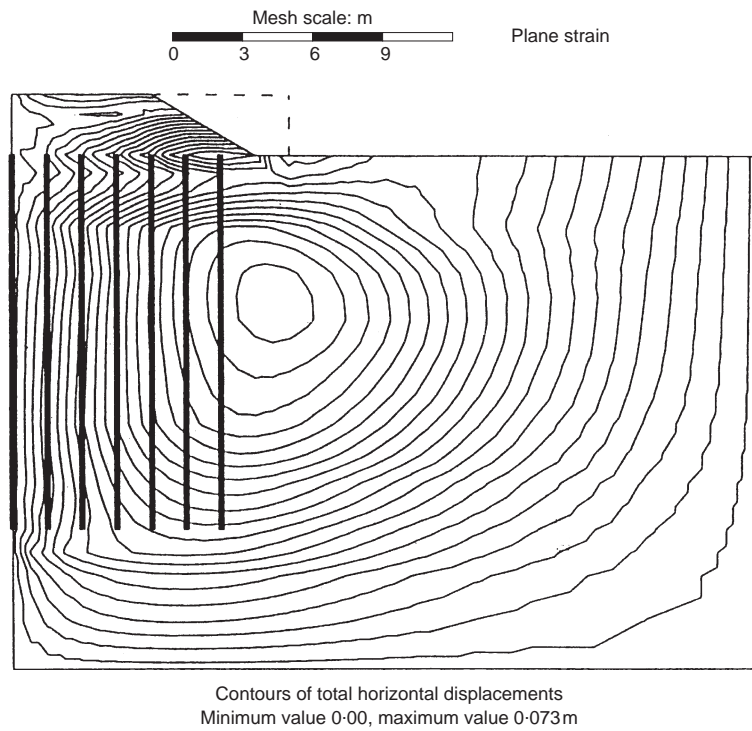


Fig. 11. Contours of lateral displacement for 2.5 m high embankment—undrained analysis

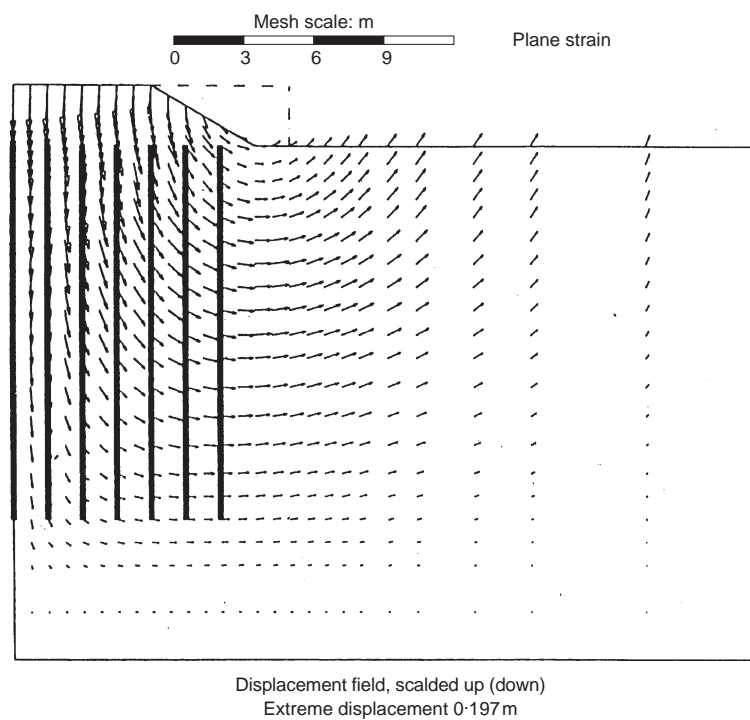


Fig. 12. Displacement field for 2.5 m high embankment—undrained analysis

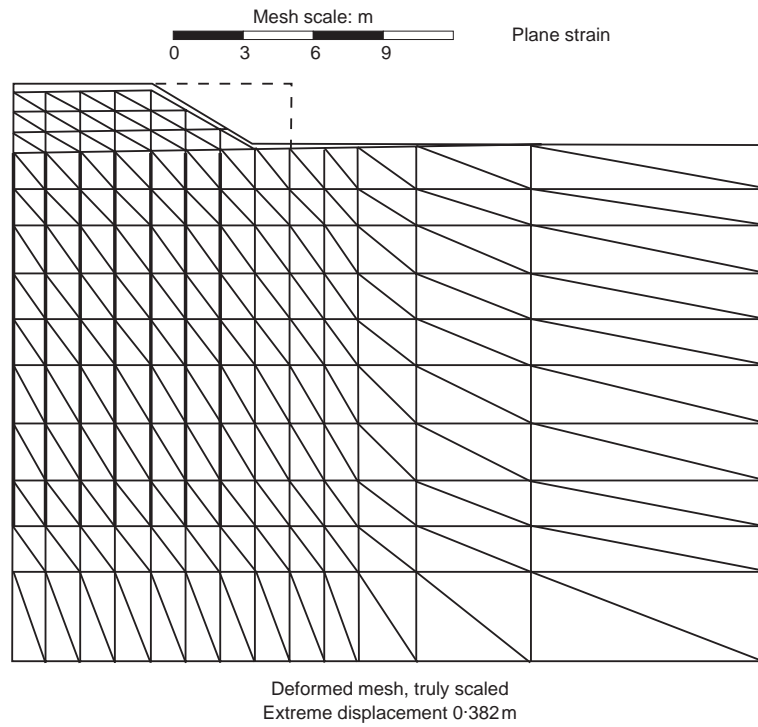


Fig. 13. Deformed mesh for 2.5 m high embankment on treated ground—fully drained analysis

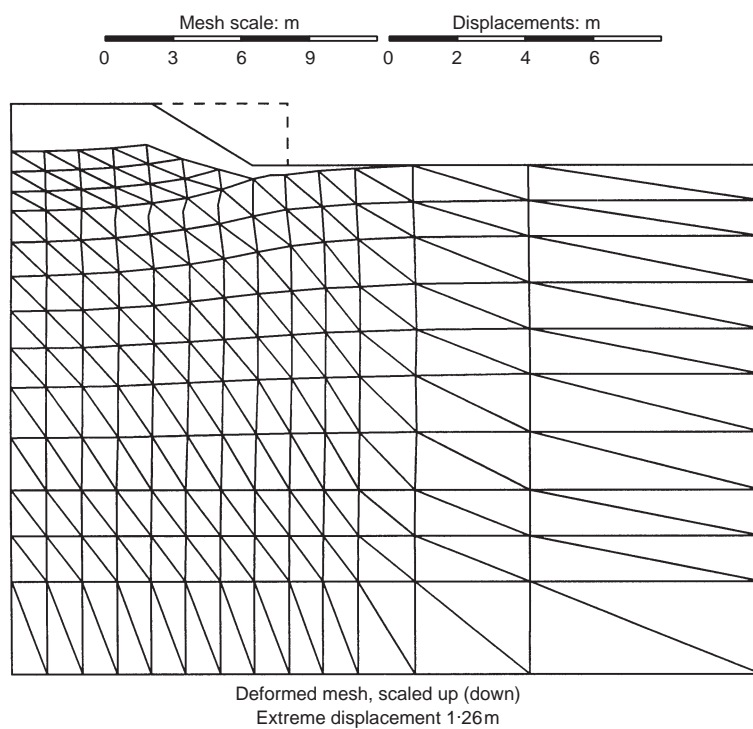


Fig. 14. Deformed mesh for 2.5 m high embankment on undrained untreated ground—fully drained analysis

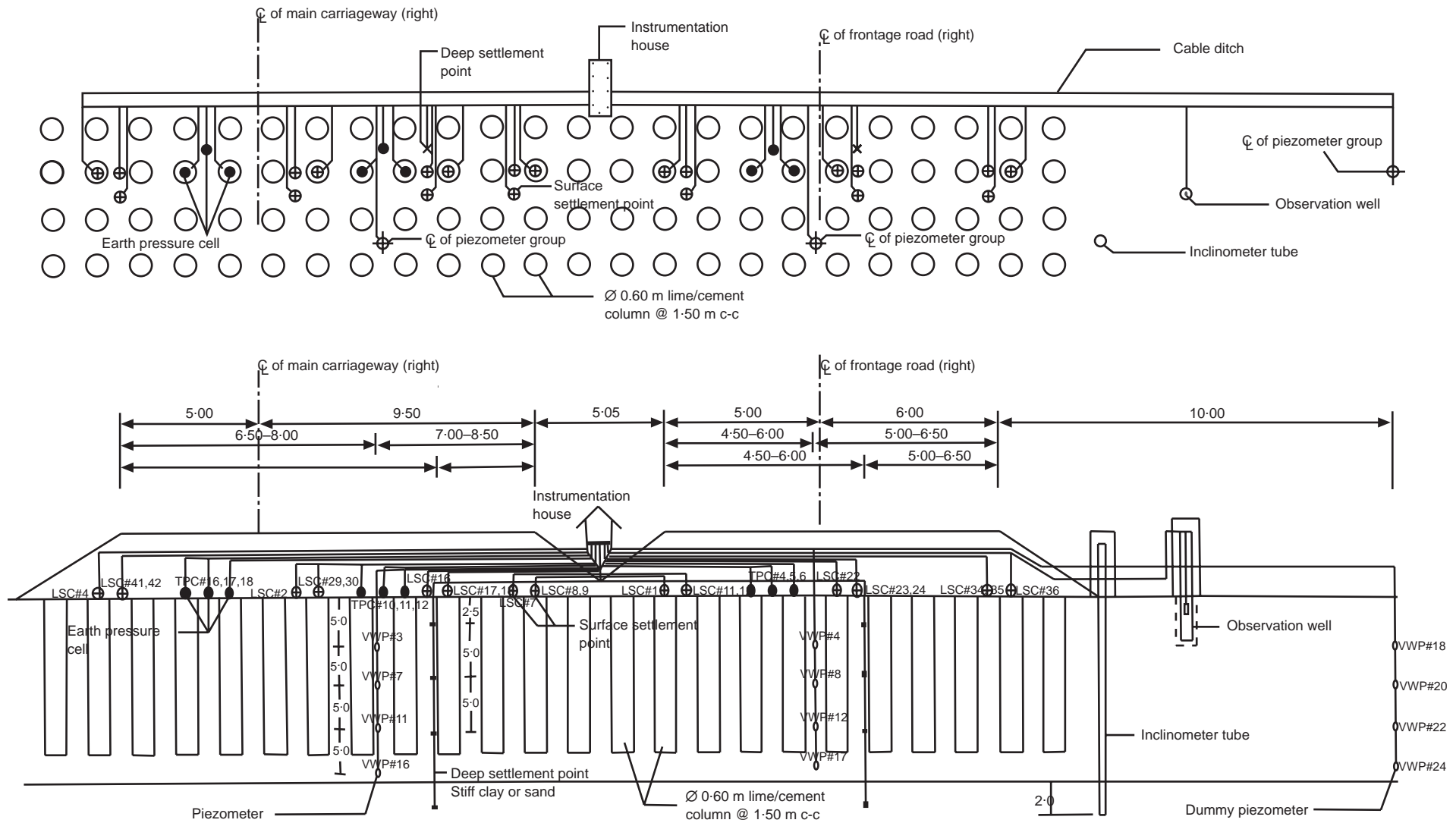


Fig. 15. Plan and cross-section of instrumentation at km 29 + 992 (not to scale) (dimensions in m)

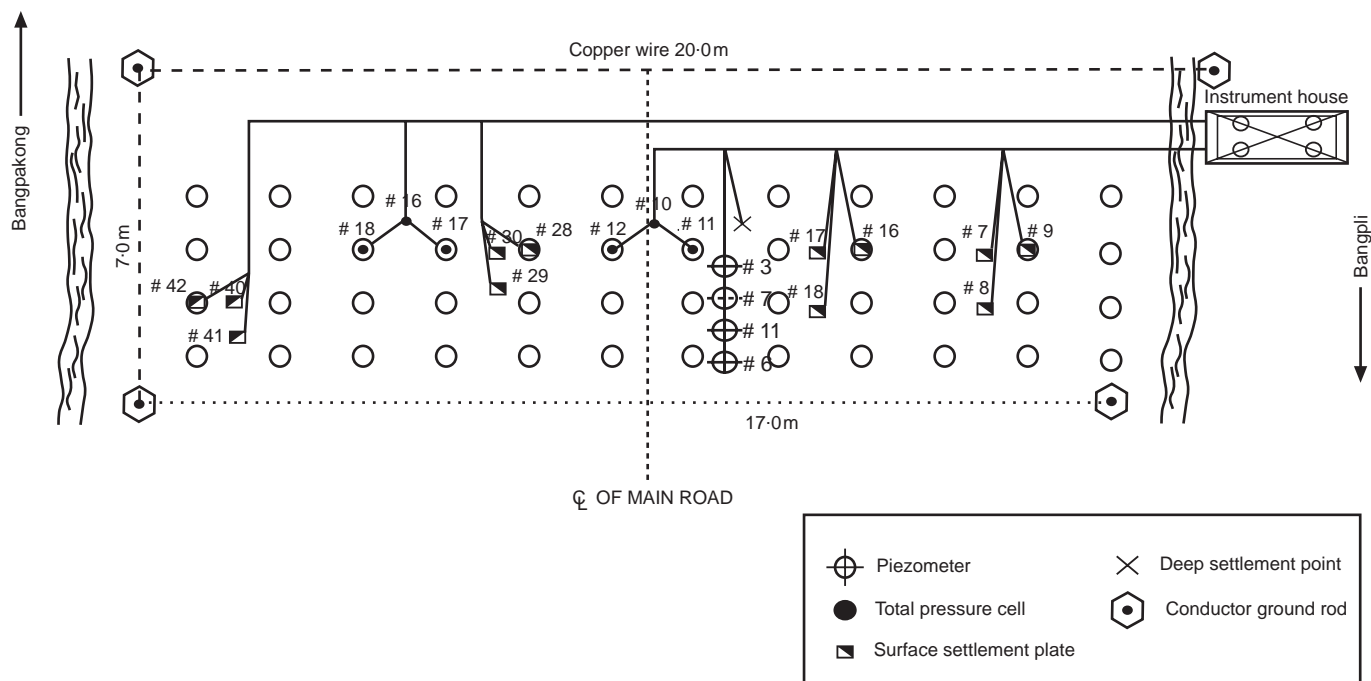


Fig. 16. Installation plan of instrumentation at main road, km 29 + 992 (not to scale)

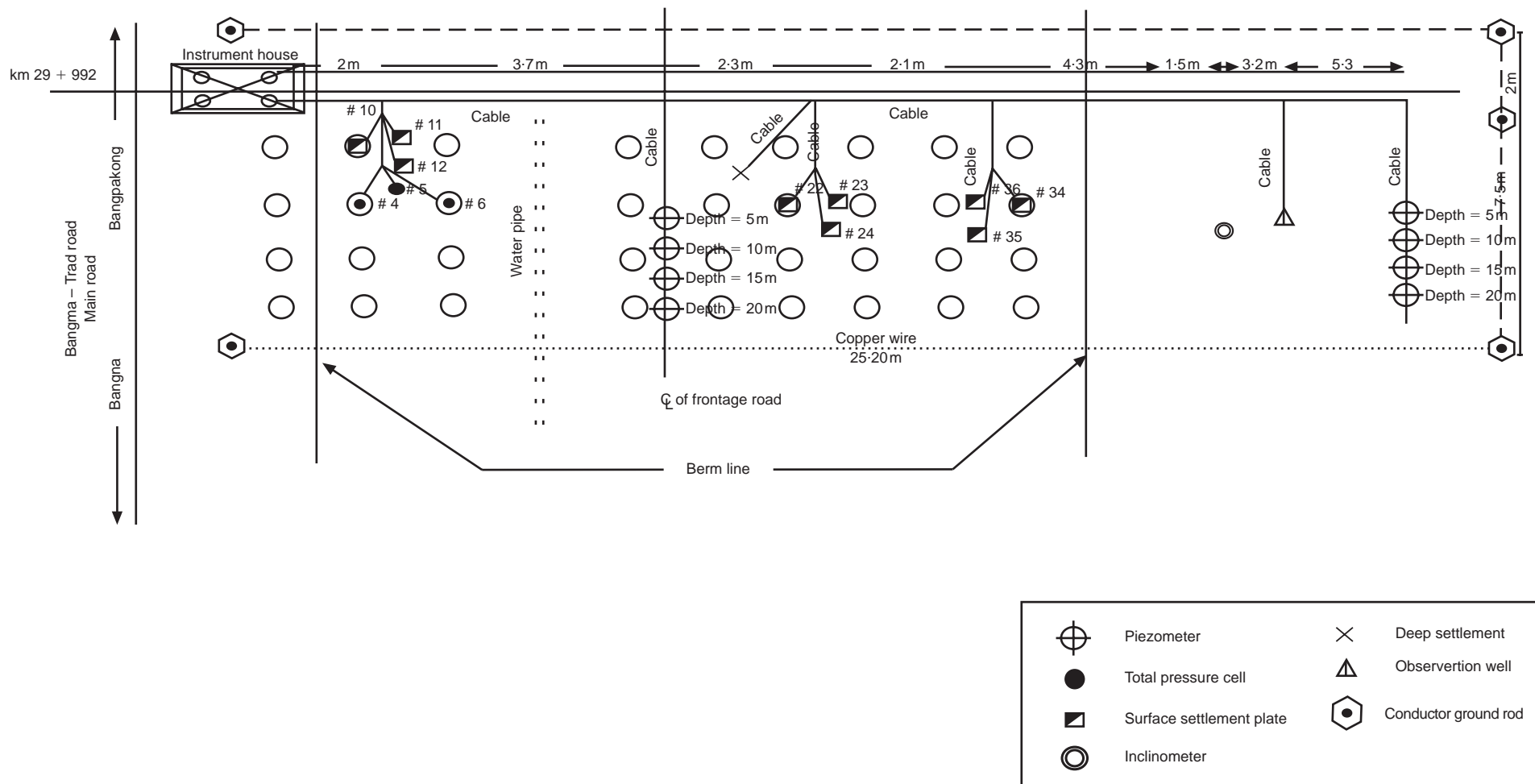


Fig. 17. Installation plan of instrumentation at frontage road, km 29 + 992 (FR3) (not to scale)

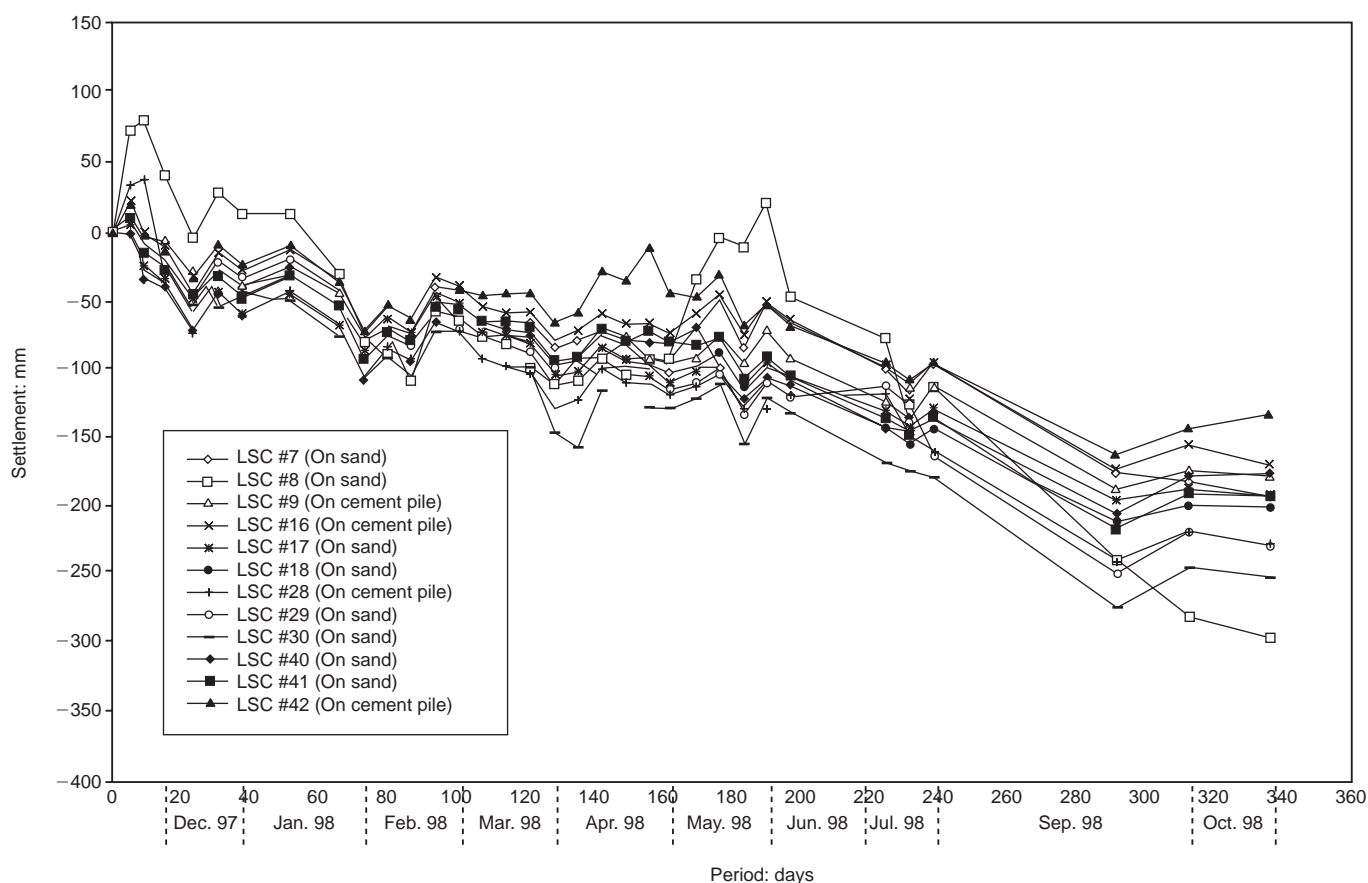


Fig. 18. Settlement cell graph at station 29 + 992 (MR3)

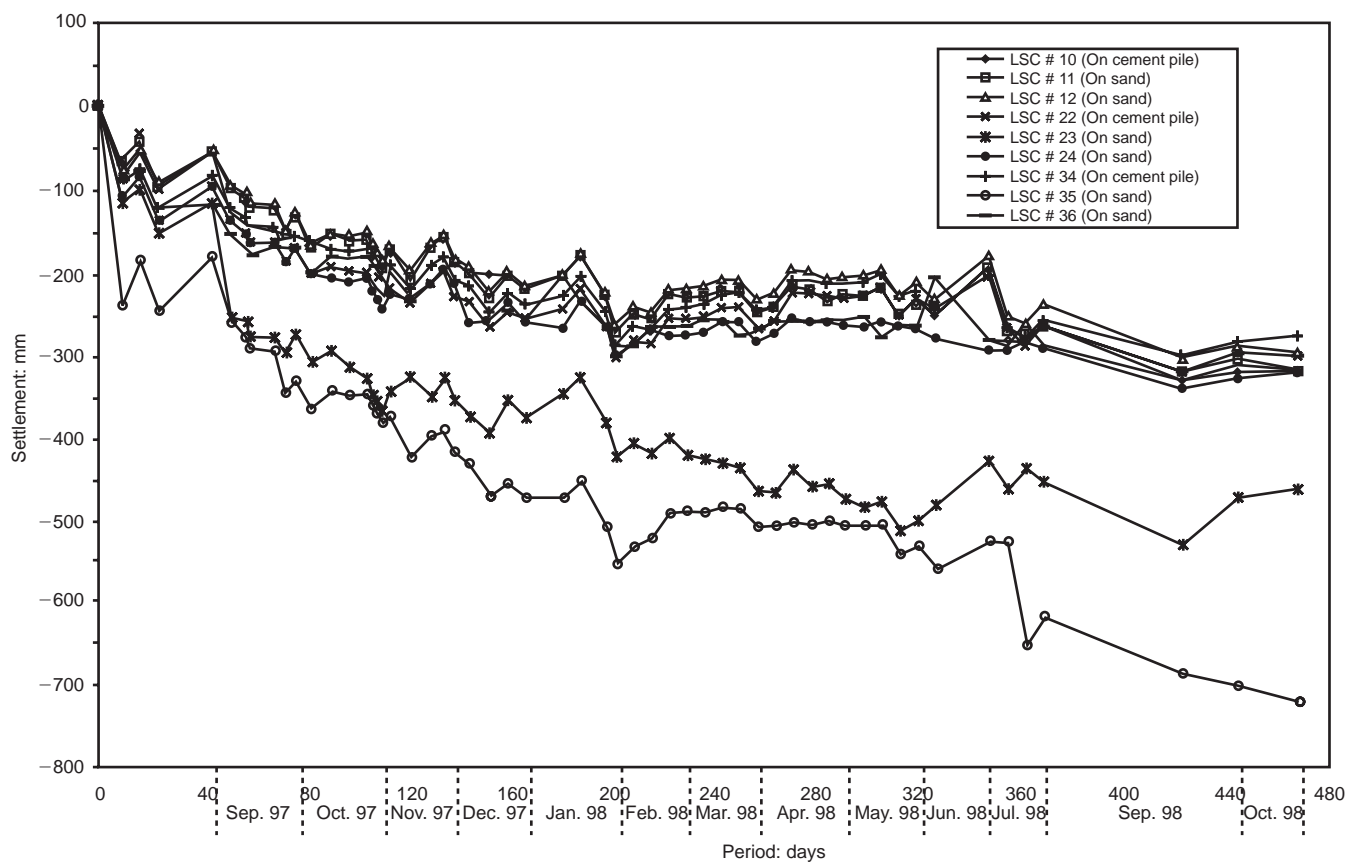


Fig. 19. Settlement cell graph at station 29 + 992 (FR3)

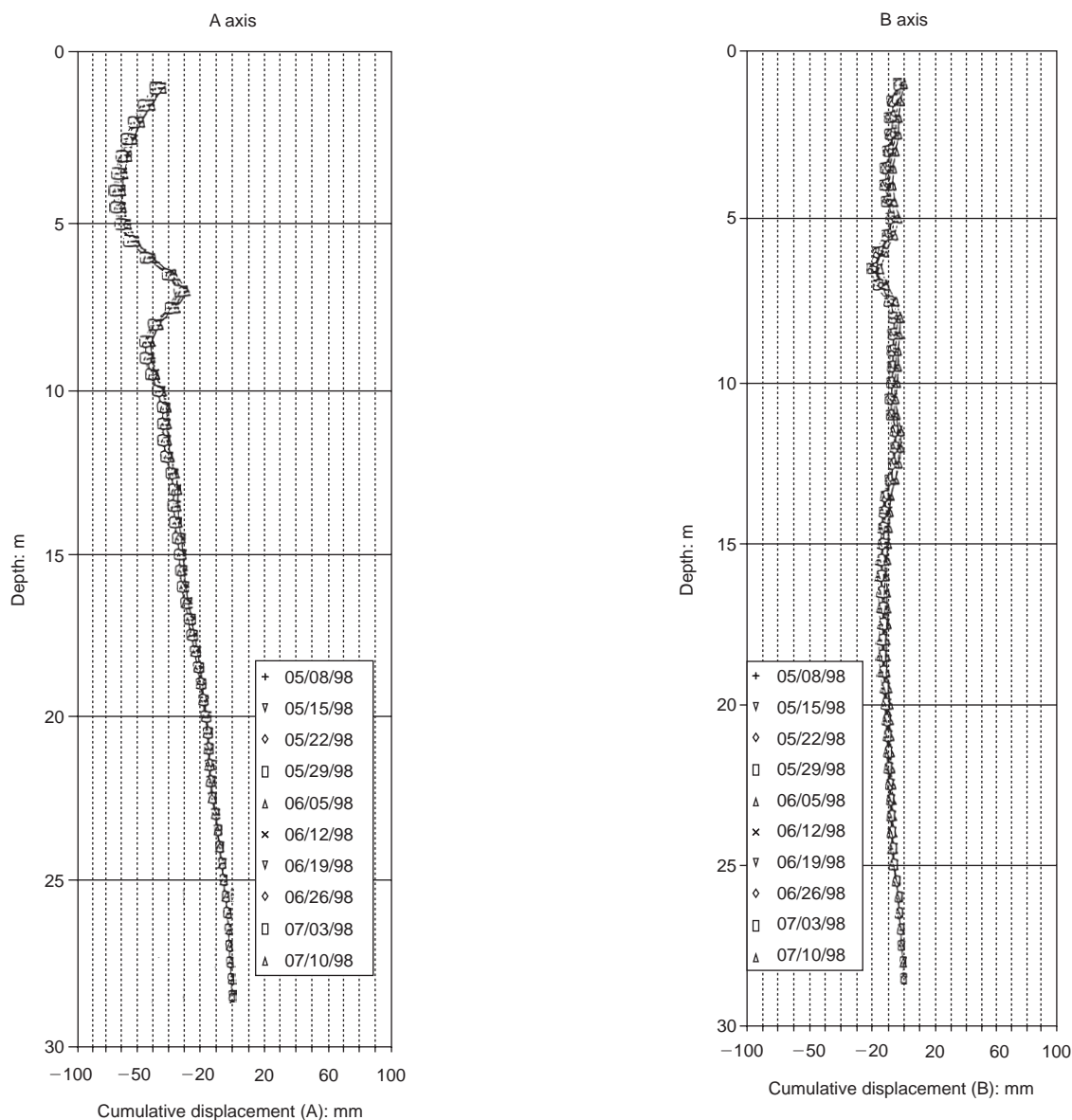


Fig. 20. Observed lateral movements from inclinometer readings in Bangna–Bangpakong Highway, km 29 + 992 (initial measurement 06/18/97)

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Discussion contributions on this paper should reach the editor by 28 April 2000